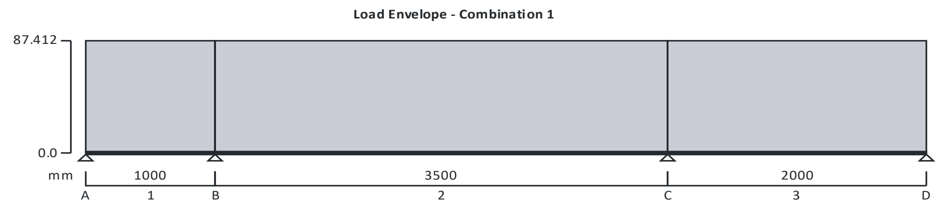
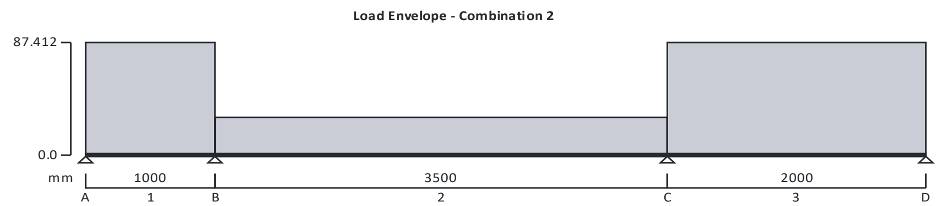
PlanningApplications.com Summer House, Upper Court Road Woldingham SURREY CR3 7BF support@planningapplications.com 07922 148 701	Project Loft Conversion - BEAM 2 - Support Floor Timbers & Stud				Job no. 7456-233	
	Calcs for Mr Andrew Rose				Start page no./Revision 1	
	Calcs by SB	Calcs date 18/05/2023	Checked by SB	Checked date 18/05/2023	Approved by DB	Approved date 18/05/2023

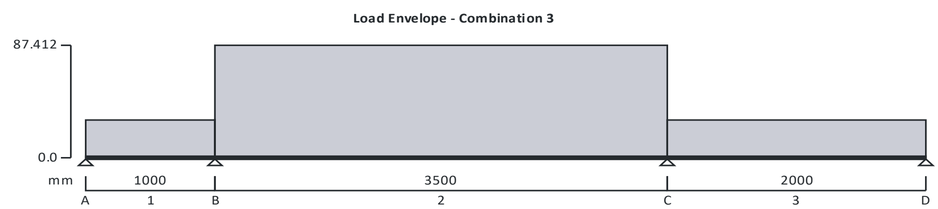
STEEL BEAM ANALYSIS & DESIGN (BS5950)

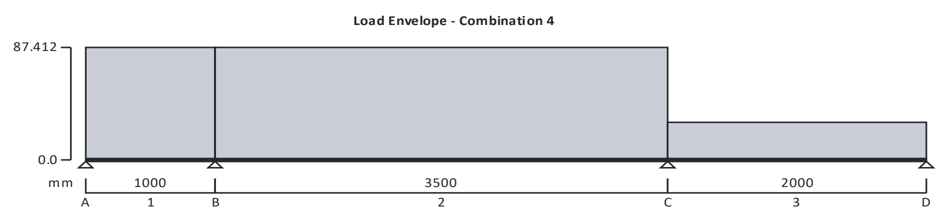
In accordance with BS5950-1:2000 incorporating Corrigendum No.1

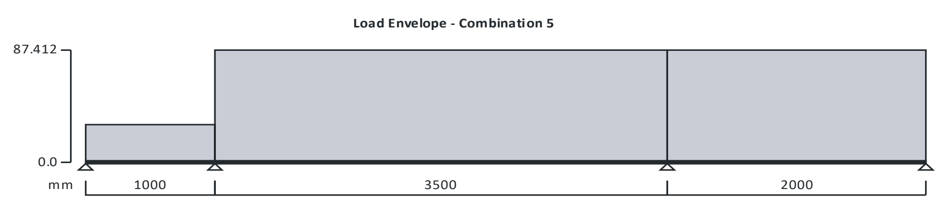
TEDDS calculation version 3.0.07

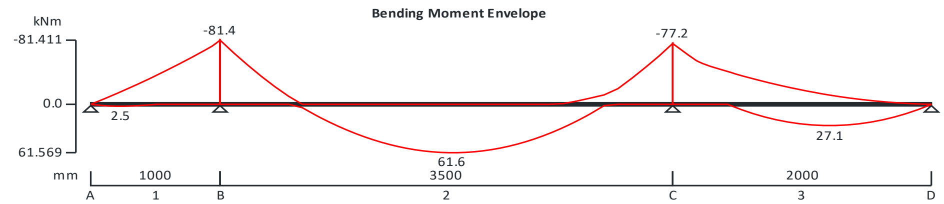


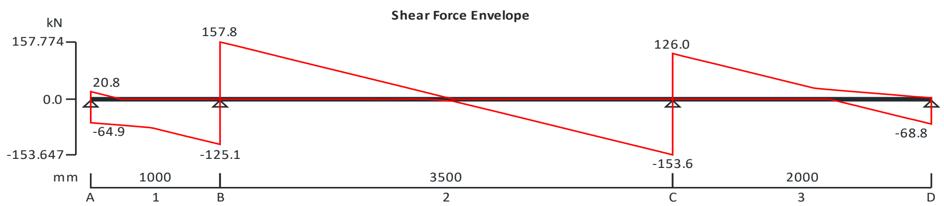












Support conditions

Support A	Vertically restrained Rotationally free
Support B	Vertically restrained Rotationally free
Support C	Vertically restrained Rotationally free
Support D	Vertically restrained Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1 Dead full UDL 29 kN/m Imposed full UDL 29 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40 Imposed × 1.60
		Dead × 1.40 Imposed × 1.60
	Support B	Dead × 1.40 Imposed × 1.60
		Dead × 1.40 Imposed × 1.60
	Support C	Dead × 1.40 Imposed × 1.60
		Dead × 1.40 Imposed × 1.60
	Support D	Dead × 1.40 Imposed × 1.60

Load combination 2	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60
		Dead × 1.00
	Support C	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support D	Dead × 1.40
		Imposed × 1.60
Load combination 3	Support A	Dead × 1.00
		Dead × 1.00
	Support B	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support C	Dead × 1.40
		Imposed × 1.60
		Dead × 1.00
	Support D	Dead × 1.00
Load combination 4	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support C	Dead × 1.40
		Imposed × 1.60
		Dead × 1.00
	Support D	Dead × 1.00
Load combination 5	Support A	Dead × 1.00
		Dead × 1.00
	Support B	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support C	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40

		Imposed × 1.60
	Support D	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **61.6** kNm	M_{min} = **-81.4** kNm
Maximum moment span 1	M_{s1_max} = **2.5** kNm	M_{s1_min} = **-81.4** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **2.5** kNm	$M_{s1\_seg1\_min}$ = **-23.3** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **2.1** kNm	$M_{s1\_seg2\_min}$ = **-49.8** kNm
Maximum moment span 1 segment 3	$M_{s1\_seg3\_max}$ = **0** kNm	$M_{s1\_seg3\_min}$ = **-81.4** kNm
Maximum moment span 2	M_{s2_max} = **61.6** kNm	M_{s2_min} = **-81.4** kNm
Maximum moment span 2 segment 1	$M_{s2\_seg1\_max}$ = **61.5** kNm	$M_{s2\_seg1\_min}$ = **-81.4** kNm
Maximum moment span 2 segment 2	$M_{s2\_seg2\_max}$ = **61.6** kNm	$M_{s2\_seg2\_min}$ = **-77.2** kNm
Maximum moment span 3	M_{s3_max} = **27.1** kNm	M_{s3_min} = **-77.2** kNm
Maximum moment span 3 segment 1	$M_{s3\_seg1\_max}$ = **25.1** kNm	$M_{s3\_seg1\_min}$ = **-77.2** kNm
Maximum moment span 3 segment 2	$M_{s3\_seg2\_max}$ = **27.1** kNm	$M_{s3\_seg2\_min}$ = **-17.9** kNm
Maximum shear	V_{max} = **157.8** kN	V_{min} = **-153.6** kN
Maximum shear span 1	V_{s1_max} = **20.8** kN	V_{s1_min} = **-125.1** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **20.8** kN	$V_{s1\_seg1\_min}$ = **-74.7** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **0** kN	$V_{s1\_seg2\_min}$ = **-96** kN
Maximum shear span 1 segment 3	$V_{s1\_seg3\_max}$ = **0** kN	$V_{s1\_seg3\_min}$ = **-125.1** kN
Maximum shear span 2	V_{s2_max} = **157.8** kN	V_{s2_min} = **-153.6** kN
Maximum shear span 2 segment 1	$V_{s2\_seg1\_max}$ = **157.8** kN	$V_{s2\_seg1\_min}$ = **-4.1** kN
Maximum shear span 2 segment 2	$V_{s2\_seg2\_max}$ = **4.8** kN	$V_{s2\_seg2\_min}$ = **-153.6** kN
Maximum shear span 3	V_{s3_max} = **126** kN	V_{s3_min} = **-68.8** kN
Maximum shear span 3 segment 1	$V_{s3\_seg1\_max}$ = **126** kN	$V_{s3\_seg1\_min}$ = **0** kN
Maximum shear span 3 segment 2	$V_{s3\_seg2\_max}$ = **38.6** kN	$V_{s3\_seg2\_min}$ = **-68.8** kN
Deflection segment 3	δ_{max} = **3** mm	δ_{min} = **0.2** mm
Deflection span 1 segment 3	δ_{s1_max} = **0** mm	δ_{s1_min} = **0.2** mm
Deflection span 2 segment 3	δ_{s2_max} = **3** mm	δ_{s2_min} = **0** mm
Deflection span 3 segment 3	δ_{s3_max} = **0.1** mm	δ_{s3_min} = **0.2** mm
Maximum reaction at support A	R_{A_max} = **20.8** kN	R_{A_min} = **-64.9** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **-11.1** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **-11** kN	
Maximum reaction at support B	R_{B_max} = **282.9** kN	R_{B_min} = **113.7** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **91.6** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **90.7** kN	
Maximum reaction at support C	R_{C_max} = **279.7** kN	R_{C_min} = **161.3** kN
Unfactored dead load reaction at support C	R_{C_Dead} = **93.4** kN	
Unfactored imposed load reaction at support C	$R_{C\_Imposed}$ = **92.5** kN	
Maximum reaction at support D	R_{D_max} = **68.8** kN	R_{D_min} = **-3.3** kN
Unfactored dead load reaction at support D	R_{D_Dead} = **16.4** kN	
Unfactored imposed load reaction at support D	$R_{D\_Imposed}$ = **16.3** kN	

Section details

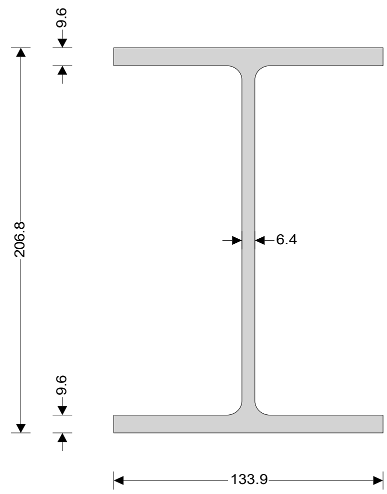
Section type	**UB 203x133x30 (BS4-1)**
Steel grade	**S275**

From table 9: Design strength p_y

Thickness of element	max(T, t) = **9.6** mm

Design strength p_y = **275** N/mm^2

Modulus of elasticity E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports plus third points

Span 2 has lateral restraint at supports plus midspan

Span 3 has lateral restraint at supports plus midspan

Effective length factors

Effective length factor in major axis K_x = **1.00**

Effective length factor in minor axis K_y = **1.00**

Effective length factor for lateral-torsional buckling $K_{LT.A}$ = **1.00**

$K_{LT.B}$ = **1.00**

$K_{LT.C}$ = **1.00**

$K_{LT.D}$ = **1.00**

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **1.00**

Internal compression parts - Table 11

Depth of section d = **172.4** mm

$d / t = 26.9 \times \varepsilon <= 80 \times \varepsilon$ Class 1 plastic

Outstand flanges - Table 11

Width of section b = B / 2 = **67** mm

$b / T = 7.0 \times \varepsilon <= 9 \times \varepsilon$ Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **157.8** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area $A_v = t \times D$ = **1324** mm^2

Design shear resistance $P_v = 0.6 \times p_y \times A_v$ = **218.4** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 2 segment 1 - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s2\_seg1\_max}), \text{abs}(M_{s2\_seg1\_min}))$ = **81.4** kNm

Reduction factor $\rho_v = [2 \times (F_{v\_s2\_seg1} / P_v) - 1]^2$ = **0.198**

Plastic modulus of shear area $S_v = t \times D^2 / 4$ = **68426** mm^3

Moment capacity high shear - cl.4.2.5.3 $M_c = \min(p_y \times (S_{xx} - \rho_v \times S_v), 1.5 \times p_y \times Z_{xx})$ = **82.7** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = 1.0 \times L_{s2\_seg1}$ = **1750** mm

Slenderness ratio $\lambda = L_E / r_{yy}$ = **55.157**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter u = **0.881**

Torsional index x = **21.493**

Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.931**

Ratio - cl.4.3.6.9 β_W = **1.000**

Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **45.280**

Limiting slenderness - Annex B.2.2 $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5}$ = **34.310**

λ_{LT} > λ_{L0} - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant α_{LT} = **7.0**

Perry factor $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.077**

Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **986.8** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **668.8** N/mm^2

Bending strength - Annex B.2.1 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **249.4** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment M_2 = **20.8** kNm

Moment at centre-line of segment M_3 = **24.9** kNm

Moment at three quarter point of segment M_4 = **51.3** kNm

Maximum moment in segment M_{abs} = **81.4** kNm

Maximum moment governing buckling resistance $M_{LT} = M_{abs}$ = **81.4** kNm

Equivalent uniform moment factor for lateral-torsional buckling

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44)$ = **0.486**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{xx}$ = **78.4** kNm

M_b / m_{LT} = **161.4** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection $\delta_{lim} = L_{s2} / 360$ = **9.722** mm

Maximum deflection span 2 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **2.983** mm

PASS - Maximum deflection does not exceed deflection limit